APPENDIX A

COMPARISON OF DESIGN METHODS

- Critique of Horner Swale Design Methodology by Gary Minton
- Comparison of Biofiltration Design Methods

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COMPARISON OF DESIGN METHODS

CRITIQUE OF HORNER SWALE DESIGN METHODOLOGY BY GARY MINTON

Minton's Method uses the approach by Horner but with certain differences. Minton believes that the primary determinant of performance is surface area, rather than length of the biofilter, as indicated by the research of Professor Barfield at the University of Kentucky. Therefore, within the constraints of proper flow distribution at the end of the biofilter, the biofilter can be configured as desired to fit the site.

Also, since performance is a function of detention time, the area of the biofilter should be increased with increasing slope, reflecting that higher velocity and shorter detention time with increasing slope. However, Manning's Equation, does the opposite; the channel narrows with increasing slope and with a constant length of 200 feet, the biofilter surface area is decreased rather than increased as the slope is increased.

Minton's Method also is based on the belief that a Manning's n of 0.10 is too low for the conditions of interest; research with shallow sheet flow in thick grasses suggests the n value should be somewhere between 0.20 to 0.60.

Finally, because of the considerable uncertainty of the effect of filter geometry and highly variable turf grass quality on performance, and the uncertainty about what is the appropriate value for n, Minton's Method is based on a view that it is pointless to have design engineers and plan reviewers spend time on sizing calculations. Their time is better spent on those aspects of design that relate to facility integrity, flow spreading, energy dissipation at the inlet, etc.

Given these uncertainties it seems valid to define an "average filter area" that will be satisfactory. The figure of 500 ft^2/impervious acre is based on a series of calculations using Manning's Equation for different slopes and two values of n, 0.30 and 0.40. The calculations were done for a one-acre site with the peak rate of 0.20 cfs. The areas for the different situations varied from about 260 ft^2 (n=0.30; slope=5 percent) to about 1,00 ft^2 (n=0.40; slope=1 percent). A value of 500 ft^2 was selected. The area requirement can be used for both swales and filter strips.

COMPARISON OF BIOFILTRATION PROJECT DESIGN METHODS

Summary of Approach Differences

Feature	Horner	Minton	King County
Design Basis	Approximate or exact Manning Equation	Width * Length = 500 ft^2/acre	Exact Manning Equation
Swale Shape	Parabolic or trapezoidal	Not specified	Trapezoidal or rectangular
Swale Slope	2 to 4 percent (<2 or >4 with special provisions)	2 to 5 percent (<2 or >5 with special provisions)	Design assuming 2 percent
Flow Depth	Free choice	Assumes 4 inches	1 inch urban, 4 inches rural (8 inches wetland veg.)
Manning's n	Choose based on veg. and depth (usually 0.05 to 0.1)	Assumes 0.3 to 0.4	Use 0.35
Basis for Filter Strip	Treates as shallow, rectangular swale	Not covered	Rule of thumb

Design Differences

The following are design cases:

- Contributing areas—range from 1 to 50 acres, assumed to be 100 percent impervious
- Contributing areas slopes—2 and 15 percent
- Design flow rate calculation basis—King County's Modified Rational Method, with contributing areas assumed to be square with longest travel path along side for time of concentration estimation
- Flow depths—1 and 4 inches
- Comparison made on the basis of swale top width times length (T * L) required. (Note: All methods are based on L=200 feet, or proportional enlargement of T if L if less than 200 feet.)
- Calculations made for both the Horner approximate method and the exact method for space-limited conditions.

The following are design results:

Case	Contrib. Area, Slope (Acre, %)	Q (cfs)	Flow Depth (inches)	Resultant Square Footage of Swale for each Design Method: Horner Approx.	Horner Exact	Minton	King County
1a	1, 2	0.57	1	4,740	1,446	—	11,900
1b			4	464	360	500	1,500
2a	5, 2	2.8	1	23,280	7,012	—	58,200
2b			4	2,280	1,793	2,500	6,200
3a	20, 2	9.9	1	82,320	24,867	—	205,000
3b			4	8,060	6,324	10,000	21,000
4a	50, 2	18.9	1	157,160	47,349	—	392,600
4b			4	15,380	12,072	25,000	39,600
5a	1, 15	0.57	1	4,740	1,446	—	11,900
5b			4	464	360	500	1,500
6a	5, 15	2.8	1	23,280	7,012	—	58,200
6b			4	2,280	1,793	2,500	6,200
7a	20, 15	11.2	1	93,140	28,120	—	232,600
7b			4	9,120	7,153	10,000	23,600
8a	50, 15	28.4	1	236,200	71,205	—	590,000
8b			4	23,120	18,108	25,000	59,400

Conclusions

The Minton and King County methods result in larger swales than either Horner method. With contributing area less than or equal to 5 acres the Minton and approximate Horner methods differ by less than 10 percent. In this case, the difference between the Minton and exact Horner methods is about 40 percent. The differentials grow to more than 60 percent (Minton versus approximate Horner) and more than 100 percent (Minton versus exact Horner) as the size and slope of the contributing area increase).

The King County method results in swales greater than or equal to 150 percent as large as those produced by the approximate Horner method at both flow depths. Comparing the King County and exact Horner methods, swales designed by the former method are more than seven times as large at the shallower flow depth an more than twice as large at the 4-inch depth. Velocities in the King County swales are less than 0.3 feet per second, compared to 0.4 to 1.1 feet per second in swales designed by the approximate Horner method and nearly 1.5 feet per second in those designed by the exact Horner method. These large differences are because of the stipulation by King County that n=0.35 be used in design. The limit in depth to 1 inch in urban areas would also make swales much larger than they would have to be.

The 200-foot-long I-5 swale that was the site of the original biofiltration work drained an area of 1.2 acres. For that area and flow at 4 inches depth, the approximate Horner method gives T=2.8 feet, which is approximately the width of that swale. The King County method yields T=9 feet at the same depth and length. The actual swale produced consistent 80 percent removals of total suspended solids and lead and 60 percent reductions of copper and zinc. Therefore, the very large swale designed by the King County method is clearly not needed to achieve high treatment efficiencies.

APPENDIX B

DESIGN DETAILS FOR H-FLUME AND FLOW SPLITTER

- 48th Avenue W Bioswale Design
- Description of H-Flume Constructions and Installation
- Flow Splitting Methodology—48th Avenue W Bioswale

APPENDIX B

DESIGN DETAILS FOR H-FLUME AND FLOW SPLITTER

48TH AVENUE W BIOSWALE DESIGN

Design Assumptions

Depth Y:	3 inches=0.25 feet
Manning's n:	0.07
Side slope:	3 horizontal:1 vertical
Longitudinal slope:	s=3 percent
Design storm:	2 years, 24-hour storm

Hydrologic Computation

Total precipitation for the 2-year, 24-hour design storm:	1.5 inches (Figure 3.5.1c, King County Surface Water Design Manual)
Total drainage area:	A=15.6 acres (calculated)
Total pervious area:	A_p=9.1 acres
Total impervious area:	A_i=6.5 acres
Estimated time of concentration:	t_c=25 minutes
Runoff estimation using Soil Conservation Service (SCS) method:	CN (pervious) 77
	CN (impervious) 98 (King County Surface Water Design Manual, Table 3.5.2B)
From the hydrograph (Figure B-1):	peak discharge (Q - peak)=1.72 cfs

Hydraulic Design

Equation 1:

$$V = \frac{1.49}{n} R^{2/3}\ s^{1/2} \text{ (Manning's Velocity Equation)}$$

Where:

V = velocity in (fps)

n = Manning's n (dimensionless), channel roughness factor

R = hydraulic radius (feet)

s = longitudinal slope

Q = AV

Equation 1 is multiplied by A on both sides

$$Q = \frac{1.49}{n} A\, R^{2/3}\ s^{1/2}$$

Manning's equation for a trapezoidal cross-section can be simplified as:

$$b = \frac{Q * n}{1.486 * Y^{1.667} * s^{1/2}} - ZY$$

$$b = \frac{1.72 * 0.07}{1.486 * (0.25)^{1.667} * (0.03)^{1/2}} - 3 * 0.25$$

b = 3.96 feet

selecting b = 4 feet

Check for Velocity

Area—A:

$$A = bY + ZY^2$$

$$A = 4 * 0.25 + 3 * (0.25)^2$$

A = 1.1875 square feet

Velocity—V:

$$\frac{Q}{A} = \frac{1.72}{1.1875} = 1.45\text{fps}$$

Actual velocity V is less than the maximum design velocity (1.5 fps).

Because the actual velocity and the maximum permissible velocity are very close, the bottom width is increased by one additional foot to reduce the actual velocity. Therefore, the bottom width to be used is 5 feet.

$$A = 5 * 0.25 + 3 * (0.25)^2$$

$$A = 1.4375 \text{ square feet}$$

$$V = \frac{Q}{A} = \frac{1.72}{1.4375} = 1.2 < 1.5 \text{ fps} \quad \text{o.k.}$$

Because storm events larger than 2-year, 24-hour storms will bypass the swale, checking for stability for the 100-year, 24-hour storm will not be required.

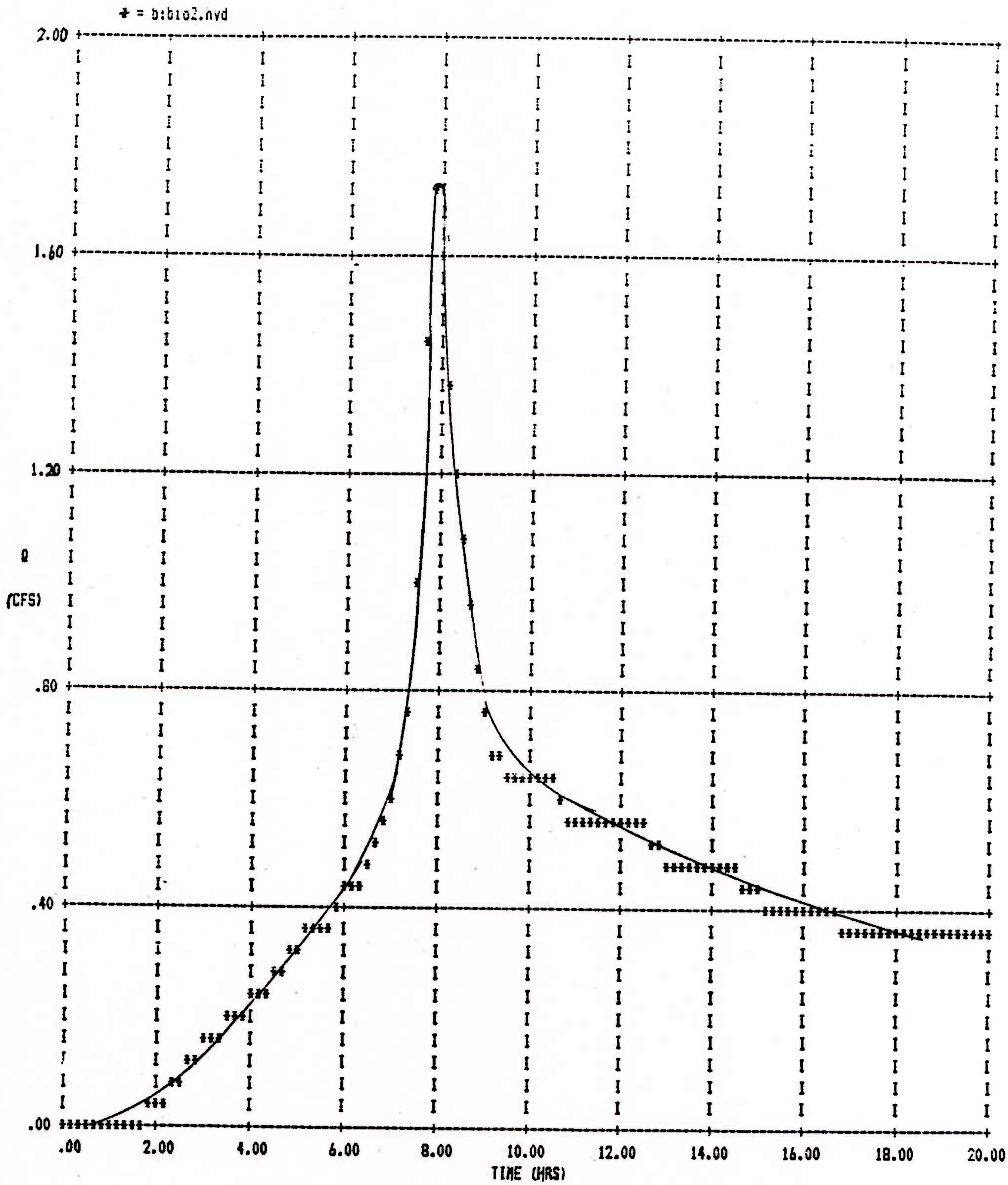


Figure B-1. Hydrograph Plot for 2-Year, 24-Hour Storm Event

DESCRIPTION OF H-FLUME CONSTRUCTION AND INSTALLATION

Inflow and outflow at the study site were measured using two standard H-flumes with a vertical range of 0 to 1.5 feet, and a discharge range of 0 to 5.42 cubic feet per second. The flumes and approach channels were constructed of varnished 0.75-inch AC plywood using dimensions from Leupold and Stevens (1987). The trailing edges of the flumes were faced with aluminum plate to provide consistent flow control surfaces.

Each flume was fitted with a 4-inch by 2.5-foot ABS stilling well to allow for the installation of water-level measuring equipment. The stilling wells were connected to the flumes at the specified measuring point using approximately 0.5 feet of 0.5-inch internal-diameter (I.D.) plastic fuel-line tubing.

The inflow to the upstream flume was through a 1.0-foot I.D. smooth plastic pcv pipe which protruded approximately 0.2 feet through the upstream end wall of the flume. The flume was set so that the approach section and control structure floor had a slope of less than 0.01. A piece of aluminum sheet approximately 1-foot by 2.5-feet weighted with 6-inch quarry spall was placed beneath the flume outfall as an energy dissipater.

The downstream flume was installed directly in the swale with plywood cut off walls on either side of the approach opening (Figure B-2). Discharge beneath the flume was prevented through the installation of a filter fabric apron nailed to the approach section floor and carefully tucked beneath the adjacent sod. As with the upstream flume, the approach and control were set so that the floor had little or no slope.

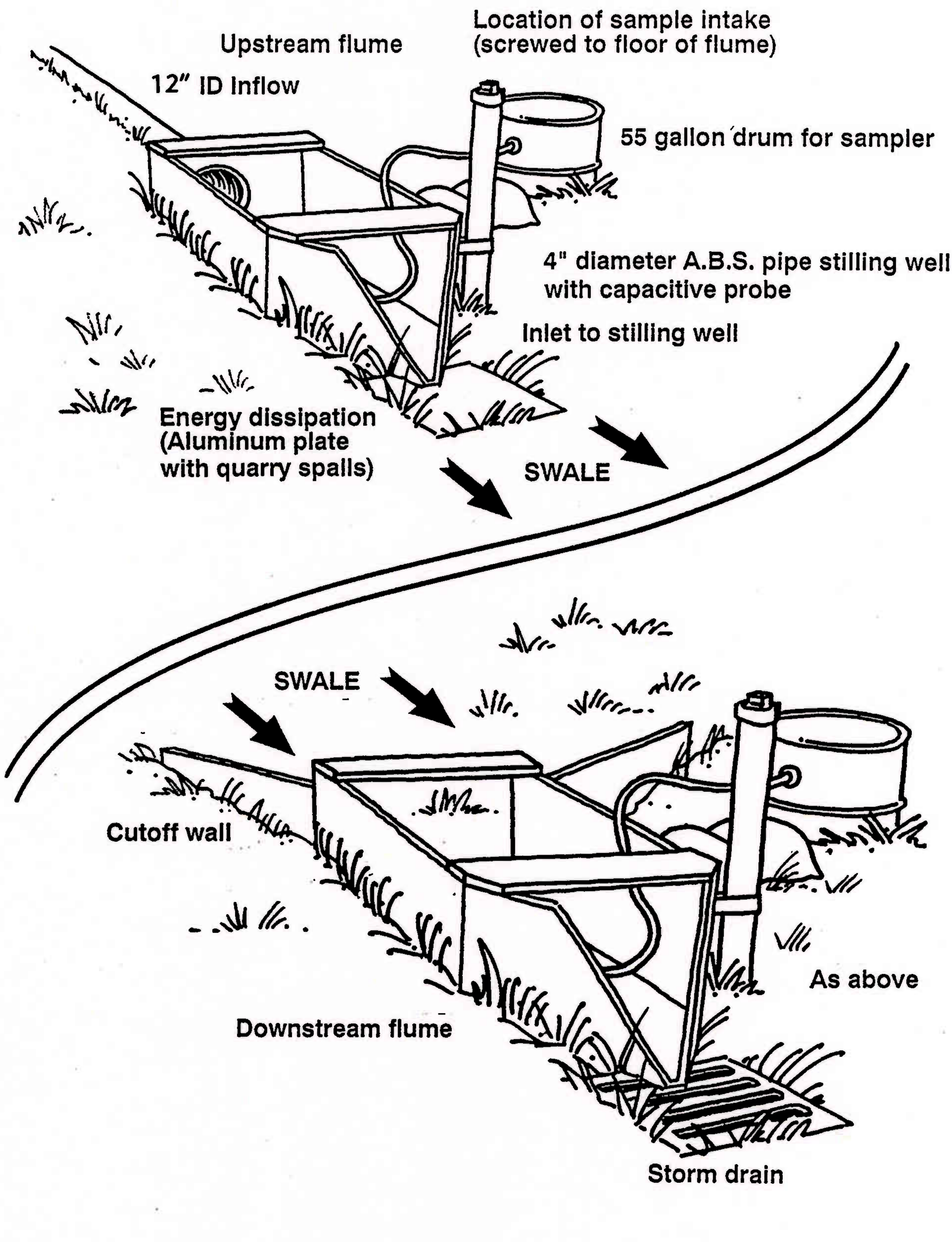
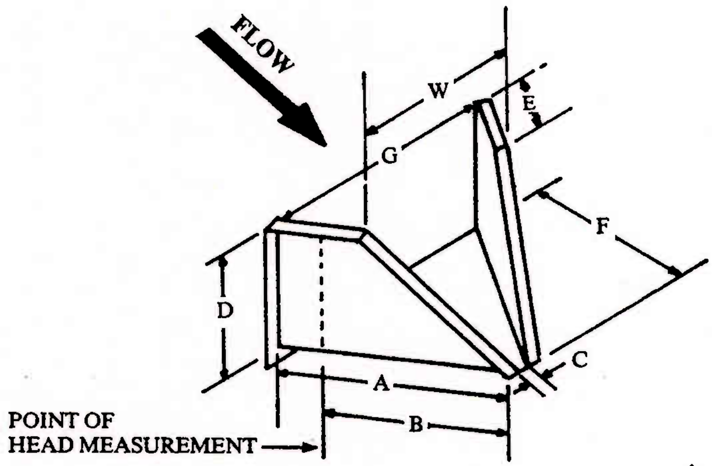


Figure B-2. Placement of H-Flume in Swale

The following exerpts and diagrams are from *Stevens Water Resources Data Book*, 1987.

> In the mid 1930s, there was a need to measure flows, with a reasonable degree of accuracy, from small agricultural watersheds and experimental plots. To meet this need, the U.S. Department of Agriculture developed the H-type flume. There are three types of H-flumes: HS flumes to measure small flows having maximum flow rates from 0.08 to 0.82 cfs; HL flumes for large flows with maximum flow rates from 20.7 to 117.0 cfs; and H flumes, which are probably used more frequently, for the mid-range with maximum flow rates from 0.35 to 31.0 cfs.
>
> The design of H-type flumes uses features of both flumes and weirs. The bottom is flat and unobstructed like a flume so silt and sediment will pass through more freely, However, the flow is controlled so it can be measured, by discharging through a sharp-edged opening like a weir, that is cut at an angle sloping back against the oncoming flow. In fact, H-type flumes are more weir than flume. Flume size is determined by the depth or height of the flume at its entrance-dimension D in Figure B-3.
>
> The approach channel for an H-type flume should be rectangular with similar depth and width as the flume and a length of 3 to 5 times the depth of the flume. Discharge through the downstream end should spill freely. The bottom of the flume should be level from intake to outflow (Figure B-4).



DIMENSIONS IN FEET FOR H FLUMES

A	HS	0.63	0.79	0.95		1.26	1.58						
	H		0.81		1.22		1.62	2.43	3.25	4.06	4.87		
	HL								4.24	5.30	6.36	7.42	8.48
B	HS	0.42	0.53	0.63		0.84	1.05						
	H		0.63		0.95		1.26	1.89	2.52	3.15	3.79		
	HL							2.65	3.54	4.42	5.30	6.19	7.07
C	HS	0.02	0.025	0.03		0.04	0.05						
	H		0.05		0.075		0.10	0.15	0.20	0.25	0.30		
	HL							0.30	0.40	0.50	0.60	0.70	0.80
E	HS	0.42	0.53	0.63		0.84	1.05						
	H		0.36		0.54		0.72	1.08	1.44	1.80	2.16		
	HL								1.41	1.77	2.12	2.47	2.83
F	HS	0.60	0.75	0.90		1.20	1.50						
	H		0.68		1.01		1.35	2.03	2.70	3.38	4.05		
	HL							2.25	3.00	3.75	4.50	5.25	6.00
G	HS	0.42	0.53	0.63		0.84	1.05						
	H		0.95		1.43		1.90	2.85	3.80	4.75	5.70		
	HL								6.40	8.00	9.60	11.20	12.80
W	HS	0.15	0.19	0.23		0.31	0.38						
	H		0.55		0.83		1.10	1.65	2.20	2.75	3.30		
	HL								4.4	5.5	6.6	7.7	8.8

MAXIMUM CAPACITY OF FLOW THROUGH H FLUMES (in CFS)

	D — Flume Size In Feet											
	0.4	0.5	0.6	0.75	0.8	1.0	1.5	2.0	2.5	3.0	3.5	4.0
HS flume	0.085	0.14	0.23		0.47	0.82						
H flume		0.35		0.97		1.99	5.49	11.3	19.7	31.1		
HL flume								20.7	36.2	57.0	83.9	117.0

Figure B-3. H-Flume Dimensions and Capacity Flow

HEAD FT.	CFS	MGD	HEAD FT.	CFS	MGD	HEAD FT.	CFS	MGD	HEAD FT.	CFS	MGD	HEAD FT.	CFS	MGD	HEAD FT.	CFS	MGD
0.01	.0000	.0000	0.26	.1183	.0765	0.51	.4730	.3057	0.76	1.130	.7303	1.01	2.140	1.383	1.26	3.590	2.320
0.02	.0011	.0007	0.27	.1275	.0824	0.52	.4930	.3186	0.77	1.163	.7516	1.02	2.190	1.415	1.27	3.660	2.365
0.03	.0023	.0015	0.28	.1371	.0886	0.53	.5140	.3322	0.78	1.197	.7736	1.03	2.240	1.448	1.28	3.730	2.411
0.04	.0039	.0025	0.29	.1470	.0950	0.54	.5350	.3458	0.79	1.231	.7956	1.04	2.300	1.486	1.29	3.800	2.456
0.05	.0057	.0037	0.30	.1570	.1015	0.55	.5570	.3600	0.80	1.270	.8208	1.05	2.350	1.519	1.30	3.870	2.501
0.06	.0078	.0050	0.31	.1680	.1086	0.56	.5790	.3742	0.81	1.300	.8402	1.06	2.400	1.551	1.31	3.940	2.546
0.07	.0103	.0067	0.32	.1790	.1157	0.57	.6010	.3884	0.82	1.340	.8660	1.07	2.450	1.583	1.32	4.010	2.592
0.08	.0131	.0085	0.33	.1910	.1234	0.58	.6240	.4033	0.83	1.380	.8919	1.08	2.500	1.616	1.33	4.080	2.637
0.09	.0164	.0106	0.34	.2030	.1312	0.59	.6480	.4188	0.84	1.410	.9113	1.09	2.560	1.655	1.34	4.150	2.682
0.10	.0200	.0129	0.35	.2150	.1390	0.60	.6720	.4343	0.85	1.450	.9371	1.10	2.610	1.687	1.35	4.220	2.727
0.11	.0237	.0153	0.36	.2280	.1474	0.61	.6970	.4505	0.86	1.490	.9630	1.11	2.670	1.726	1.36	4.300	2.779
0.12	.0276	.0178	0.37	.2410	.1558	0.62	.7220	.4666	0.87	1.530	.9888	1.12	2.730	1.764	1.37	4.370	2.824
0.13	.0319	.0206	0.38	.2550	.1648	0.63	.7470	.4828	0.88	1.570	1.015	1.13	2.780	1.797	1.38	4.450	2.876
0.14	.0365	.0236	0.39	.2690	.1739	0.64	.7730	.4996	0.89	1.610	1.041	1.14	2.840	1.835	1.39	4.520	2.921
0.15	.0414	.0268	0.40	.2830	.1829	0.65	.8000	.5170	0.90	1.650	1.066	1.15	2.900	1.874	1.40	4.600	2.973
0.16	.0467	.0302	0.41	.2980	.1926	0.66	.8270	.5345	0.91	1.690	1.092	1.16	2.960	1.913	1.41	4.680	3.025
0.17	.0523	.0338	0.42	.3140	.2029	0.67	.8550	.5526	0.92	1.730	1.118	1.17	3.020	1.952	1.42	4.760	3.076
0.18	.0582	.0376	0.43	.3300	.2133	0.68	.8830	.5707	0.93	1.780	1.150	1.18	3.080	1.991	1.43	4.840	3.128
0.19	.0645	.0417	0.44	.3460	.2236	0.69	.9120	.5894	0.94	1.820	1.176	1.19	3.140	2.029	1.44	4.920	3.180
0.20	.0711	.0460	0.45	.3630	.2346	0.70	.9420	.6088	0.95	1.860	1.202	1.20	3.200	2.068	1.45	5.000	3.231
0.21	.0780	.0504	0.46	.3800	.2456	0.71	.9720	.6282	0.96	1.910	1.234	1.21	3.270	2.113	1.46	5.080	3.283
0.22	.0854	.0552	0.47	.3980	.2572	0.72	1.002	.6476	0.97	1.950	1.260	1.22	3.330	2.152	1.47	5.160	3.335
0.23	.0931	.0602	0.48	.4160	.2689	0.73	1.033	.6676	0.98	2.000	1.293	1.23	3.390	2.191	1.48	5.240	3.387
0.24	.1011	.0653	0.49	.4350	.2811	0.74	1.065	.6883	0.99	2.050	1.325	1.24	3.460	2.236	1.49	5.330	3.445
0.25	.1095	.0708	0.50	.4540	.2934	0.75	1.097	.7090	1.00	2.090	1.351	1.25	3.520	2.275	1.50	5.420	3.503

Figure B-4. Discharge of 1.5-Foot H-Flume

FLOW SPLITTING METHODOLOGY—48TH AVENUE W BIOSWALE

The testing of the hypothesis regarding the effectiveness of a 100-foot-long swale versus a 200-foot-long swale required that the flow into the swale be reduced by one half of its previous amount. The device chosen to perform this splitting of flows was the sharp-edged weir (Figure B-5).

Two weirs were installed at the same elevation inside the flow diversion manhole upstream of the swale. The inside of the manhole was modified to form a "level pool" for the routing of the flows. One weir was installed as a vertical riser pipe in a 90-degree pipe elbow. Here the horizontal end of the pipe elbow was cemented to the inside wall of the 48-inch diameter manhole and into the pipe outlet that runs to the swale. The other weir, in the form of a plate with a horizontal crest, was fastened over the street outlet. The elevation of the two weirs' crests could be made the same by observing the depth of flow over each weir crest and adjusting the riser pipe as needed.

The following describes the two different empirical formulas that were used to predict and design the flow splitting device that was installed in the manhole on 48th Avenue W in Mountlake Terrace. The device chosen is fully constrained, sharp-edged weir. This presentation compares the calculated flows, for a given height above the crest, as determined by two empirical formulas. The maximum design flow to be split is 2.0 cfs. Flow more than this amount would not be split evenly.

The first equation used is for a notched weir that comes from King County Stormwater Management Manual for a circular drain (vertical pipe) in the fully constrained condition, where all angles of the notch are 90 degrees. The following coefficients are described. All dimensions are given in feet except where noted.

$I = 1 \ldots 16$

$L = 1.5$

$P_I = 5.050 + H_I$

$H_I = I * 0.025 + 0.200$

$C_I = 3.270 + 0.400 * (H_I/P_I)$

$Q = C_I * (L - 0.2H_I) * (H_I)^{1.5}$

Where:

I = iterations

H = height of flow from sharp-edged weir crest

P = height from invert of outlet to sharp-edged weir flow height

C = empirical formula coefficient

L = total length of both sharp-edged crests, L/2 is the arc length of the circular drain edge (which is not to exceed 50 percent of the circumference)

Q = flow rate in cubic feet per second

For the sake of comparison, the water is started at a height of 0.2 feet above the sharp-edged crest and the discharges are calculated for the two types of weirs. The results are shown in Table B-1.

The second method chosen is known as the "Alternative Weir Formula" from the U.S. Bureau of Reclamation. It is used to describe the flow over the thin plate weir in covering the outlet to the street.

Where: $Q_p = CeLe(He)^{3/2}$

Q_p = discharge in cubic feet per second over the thin plate weir

Ce = a discharge coefficient

Le = L + kb

He = H + kh = H + 0.003

J = iterations

and

L = measured length of the weir crest in feet

B = width of approach channel

H = head measured above the weir crest in feet

kb = correction factor for weir length related to channel geometry

kh = correction factor for weir head of water flow

Start by providing measurements or ranges of values:

J = 1...16 B = 4.00 (L/B) = 0.38 $He_j = -H_j + 0.003$

From the graph of Figure 16, page 38, the L/B ratio yields a value of:

$kb = 0.008$ and $Le = L + kb$

Using Figure 17, page 39, to derive Ce, where:

$Ce_j = 3.220{:}0.400 * (H_j/P_j)$

The discharge equation can now be written for Q where:

$Qp_j = [Ce_j * Le * (He_j)^{1.5}]$

The values for discharge for the King County weir formula (Q_j) and the Bureau of Reclamation formula (Qp_j) are displayed with their respective water height at the weir crest (H_j, He_j) in Table B-1.

Table B-1. Weir Formula Discharge Values

H_j, He_j	Q_j	Qp_j
0.23	0.51	0.66
0.25	0.60	0.77
0.28	0.69	0.89
0.30	0.78	1.01
0.33	0.88	1.14
0.35	0.98	1.28
0.38	1.08	1.42
0.40	1.19	1.56
0.43	1.29	1.71
0.45	1.41	1.86
0.48	1.52	2.02
0.50	1.64	2.18
0.53	1.76	2.34
0.55	1.88	2.51
0.58	2.00	2.69
0.60	2.12	2.86

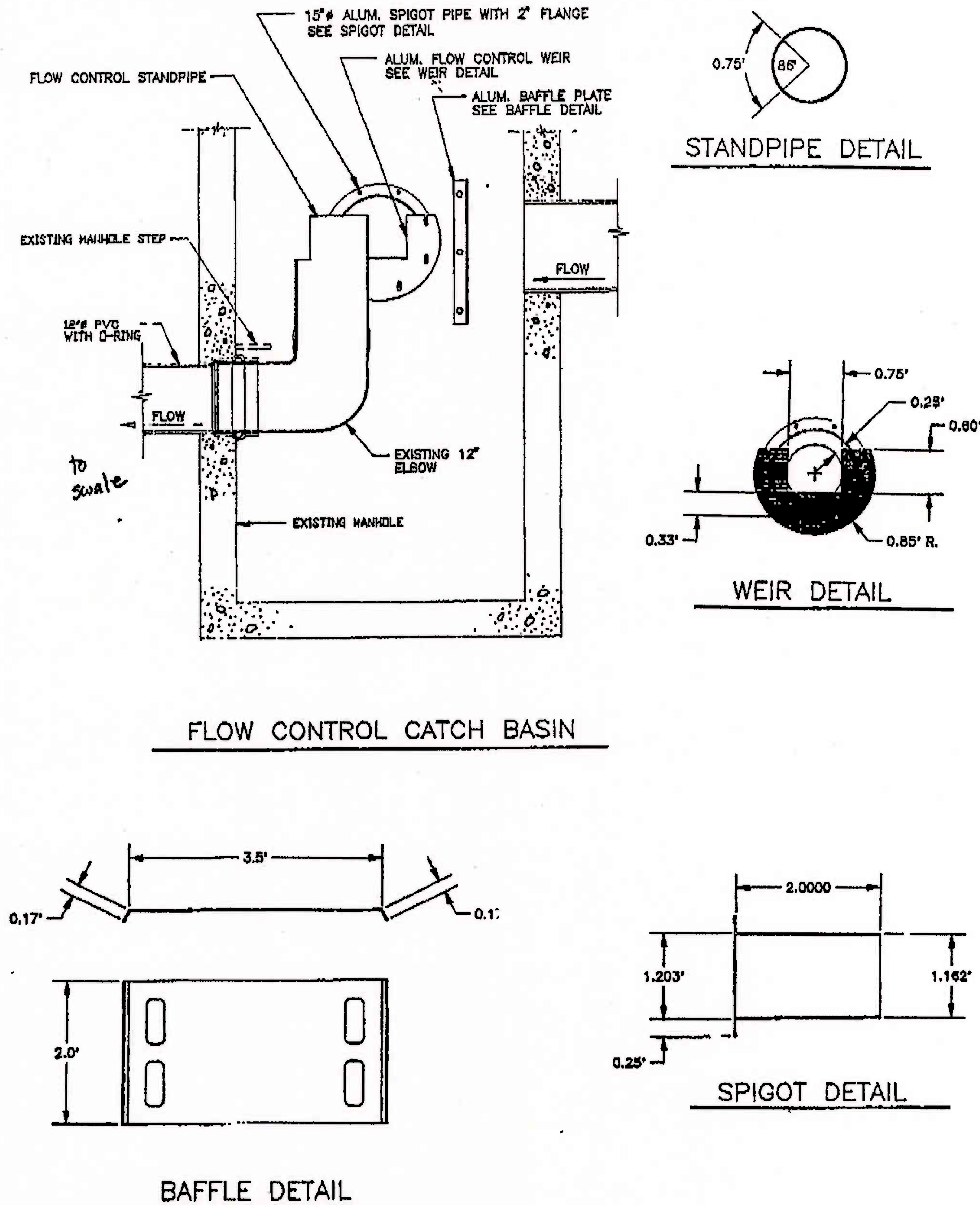


Figure B-5. Flow-Splitter Design Details